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Abstract: The Upper Mun Dam located in Nakorn Ratchasima, the Northeastern Province of Thailand, intercepts a catchment area of 454 sq. kms. It is a zone earth dam with 32.7 meters of maximum height and 880 meters in length. The foundation strata comprises alluvial deposits overlying sandstone and siltstone. The chute spillway is designed for probable maximum flood of 950 m^3/sec. and the outlet conduit is 1.84 by 1.84 meters caters for a maximum outflow of 11.2 m^3/sec. No foundation treatment was carried out except cement grouting beneath the spillway and the outlet. The dam was completed on May 1989. The maximum storage for the first year impoundment was 33.7 million m^3 with water level at the elevation 212.32 meters. The river bed elevation is 205.00 meters. In October 1990, the water level rose to nearly full reservoir level at the elevation 220.42 meters with the storage volume of 131.8 million m^3 due to heavy rain fall when concentrated seepage was noticed at two locations downstream of the toe. At one of the two locations, the leakage caused a cave-in of the down stream slope of the embankment which pluged and stopped the leakage. At the other location, concentrated leak reached its peak of about 5 m^3/sec. and resulted in progressive erosion of embankment toe. Many countermeasures to save the dam from falling were deployed. Earth moving equipments were used to push riprap as well as embankment material continuously into the leakage locations. The reservoir level was lowered through the outlet and siphonning over the spillway crest. The operation was continued for 3 weeks before the leakage was brought down to a few litre/sec. of clear water. During this incident 57 villages involving over 13,000 residents had to be evacuated. After the reservoir was emptied in a few months later, an intensive investigation program was carried out to look into the cause that lead to the leakage of the dam.

1. INTRODUCTION

The Upper Mun Dam or Mun Bon Dam located 1.7 Km. south-west of Khonburi district in Naknon Ratchasima, northeastern province of Thailand (See location Map in Fig.1). The Project was the consequence of a proposal submitted by the Royal Irrigation Department for a scheme to develop the Upper Mun and its eastern tributary the Lam Sae and entails the construction of two storage dams and canals system serving about 19,100 hectares of agriculture land. The feasibility study(1) was carried out and finished since 1970. The details design was completed in 1984 and the construction work was completed by a local contractor in 1989. During the first rainy season, the reservoir was impounded to elevation 213 m. or 39 million m^3. On October 4, 1990 the reservoir level rose from elevation 210.79 m. with storage volume of 21.8 million $m.^3$ to elevation 220.35 m. with storage volume 130.7 million m^3. on October 22, before leakage was observed.

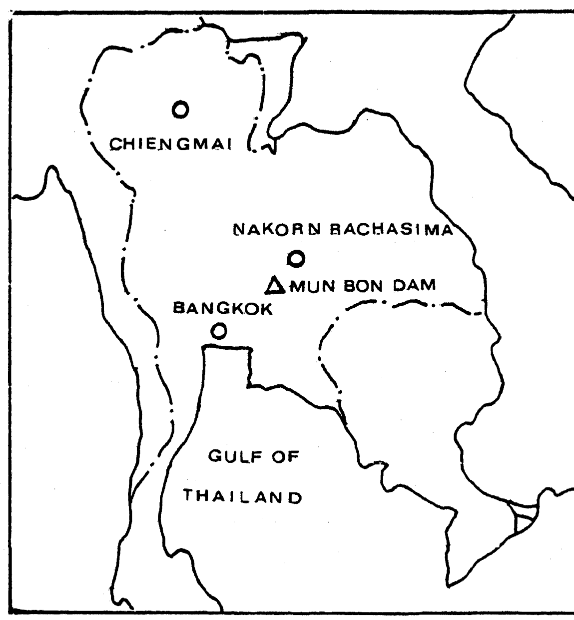

Fig. 1 Location map

Following the incident, a technical investigation committee was formed to investigate the cause of leakage. The authors were designated by RID to serve on this committee. Therefore, the views expressed in this paper definitely influenced by the results observed by the committee. However, this paper does not represent the opinion or the policy of RID.

2. SITE CONDITIONS AND EMBANKMENT

The Upper Mun Dam as designed and constructed has the following features.

The dam was founded on alluvium deposits, overlying rocks of Khorat Group comprising sandstones, siltstones, and mudstones. These rocks are exposed on the hills on either side of the valley on the dam abutments. The alluvial deposits, as much as 19 meter thick, primarily consist of clay, silty sand and sandy silt soils of low plasticity. Fig.2 depicts the geologic section along the center line of the dam.

The embankment is a modified homogeneous earthfill dam with length of 880 meter and height of 32.7 meter above excavation works. The upstream slope is covered with riprap over the operating range of the reservoir where as the downstream slope is covered with grass sodding.

An inclined chimney drain, 3 meters in horizontal width consisting of fine filter material, is provided to assure positive control of the phreatic line and prevents erosion of material from the impervious zone of the dam.

Since the dam is founded on alluvial deposits and no cutoff trench was provided except at both abutments. The underseepage water through the foundation was controlled by a sandwich drainage blanket underlying the downstream shoulder together with a row of pressure relief wells along the downstream toe of the dam. This sandwich drain consists of 0.5 meter thick of

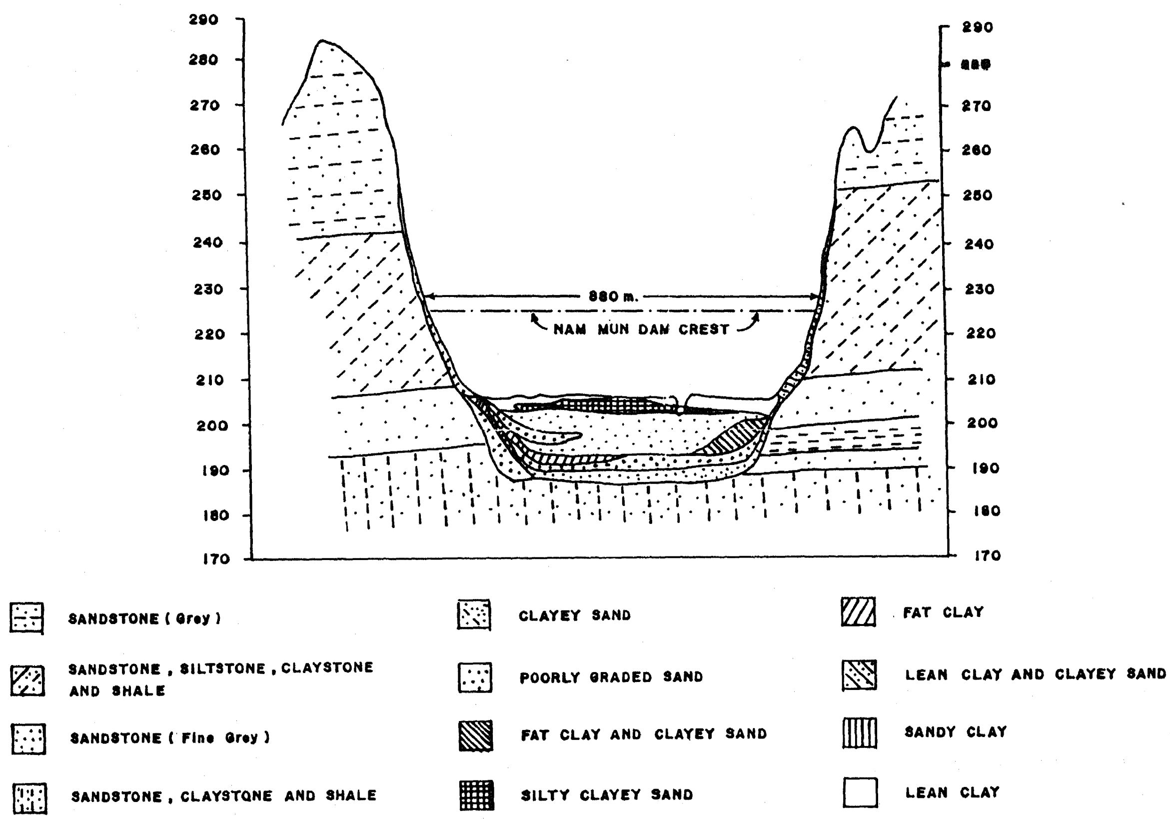

Fig. 2 Geologic section along center line of the dam[2]

coarse filter material in between fine filter layers of which the top layer is 0.5 meter thick and the lower layer is specified for a minimum thickness of 0.15 meter.

Fig.3 shows two typical cross sections of the dam. The dam crest is at Elv.+230.70 meter. The probable maximum water level is set at Elv.+228.9 m. with a capacity of 350 million m. while the normal storage level is at Elv.+221.0 meter with a capacity of 141 million m^3.

The dam consists of a service spillway situated at the right abutment which could discharge water at the rate 95 m^3/sec. for probable maximum flood and an outlet conduit 1.84 m. by 1.84 m. located at the left abutment with discharge capacity at the

the general foundation level. The upstream slope is 3H:1V for the upper 10 meters and flatten to 5H:1V over the lower section. Over the river section for a length of about 400 meter, the dam has been extended with an outer slope 15H:1V to form a partial blanket and stabilized berm on the soft alluvium materials. The downstream slope is 2.5H:1V which also flatten in the lower part to 5H:1V for the length of about 250 meters over the river section of the dam where the foundation is soft.

The zone upstream of the inclined chimney drain consists of impervious earthfill. The outer zone of the downstream shoulder consists of a random material obtained from spillway and outlet

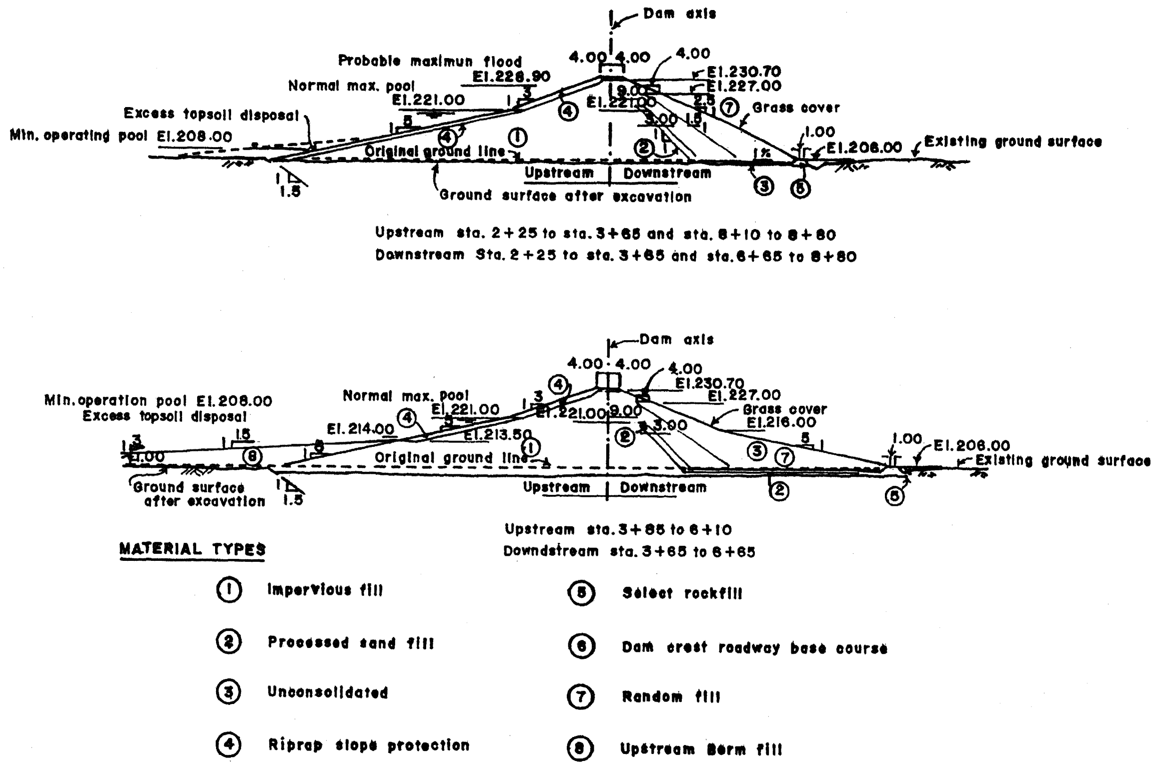

Fig. 3 Typical cross section of the dam

rate of 11.2 m^3/sec.

3. HISTORY OF IMPOUNDING

Timelines of reservoir elevations, since the commissioning of the dam until immediately following the failure incident, is shown in Fig. 4. During 1989 wet season the reservoir has only reached elevation 213 m. As may be observed from October 4 to October 23, 1990 the reservoir elevation rose rapidly from 210.79 m. to 220.35 m. as a consequence of two consecutive depression storms.

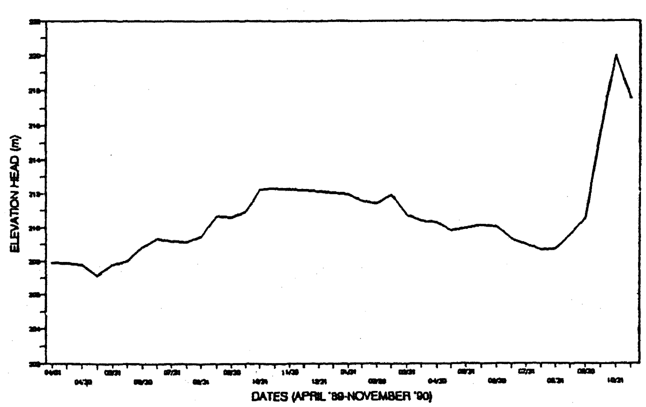

Fig. 4 Plot of Reservoir Elevation VS. time before incident

The sto'age volume, thus increased from 21.8 million m^3. to 130.7 million m^3.

4. HISTORY OF LEAKAGE

October 22, a leakage appeared at the toe of the downstream slope at STA 0+856. The hole was approximately 15 cm. in diameter. The leakage water was clouded with sediment. Rock and aggregate were dumped into the hole in order to stop the boiling. However, the attempt was not succeeded. The leakage amount increased and the hole enlarged throughout that night.

In the morning of consecutive day, another leakage larger than the first one was found on the downstream slope at STA 0+475. This latter leakage was in the dam body approximately 5 m. above the toe level. An attempt was made without success to stop the leakage throughout the night, therefore the outlet gate was opened to release the impounding water.

October 24, a cave-in was developed at the downstream slope at STA 0+475, approximately at elevation 226 m. The sinkhole was 3.5 by 6 meters and extended deep into the dam body. The collapse of the embankment, however, stopped the leakage at the second location. Concentration was then given to the leakage at the first location. Materials were dumped into the hole as water gushing out off it. The volume of leaking water was 2 to 3 m^3/sec. The effort to prevent further damage to the dam was going on throughout the night.

October 25, at day light, a vortex was seen at the upstream of the leakage point. Material from upstream part of the dam was pushed into the reservoir in order to plug the hole at the vortex. The reservoir level rose to Elv + 220.42 m. at 6 p.m. an increase in storage,despite all the emergency discharge efforts. The leakage exceeded 5 m^3/sec. before decreasing to about 1 m^3/sec. late in the evening, as much of the top part of the dam was cut and pushed into the leakage area.

October 26, early in the morning, the quantity of the leakage increased again and reached its peak of about 6 m^3/sec. at around 11 a.m . At 8.30 a.m. a tension crack was observed at the downstream slope from STA 0+800 to STA 0 +850, just above the leakage point, approximatly 3 m. below the crest. Before 9 a.m. small cracks appeared on the crest. The cracks on the slope grew and developed to a complete circular failure. By 10 a.m. movement of the sliding earth was observed (Fig.5) . Shortly before 1 p.m. a loud cracking sound occurred for about 15 minutes as the embankment collapsed and caved-in. With the continuing efforts in plugging the dam , in the late evening the leakage decreased to about 1 m^3/sec.

October 27, leakage volume was reduced to less than 1 m^3/sec. owing to the successful counter measures taken against the erosive leaking water. However , the leakage was constantly continued throughout the night.

In spite of the reduction in leakage quantity, the dam was considered unsafe, since the resevoir level was still high. With a large amount of inflow from upstream, the reservoir level was hardly reduced despite the effort to discharge 11.2 m^3/sec through the outlet. To further reduce the storage, pipe siphons over the spillway crest were used for increasing the discharge capacity.

October 28 to 31, since an amount of embankment was slowly eroded away with the leaking water, the measures taken to counter the downstream slope collapse were to fill in small amount of rock, gravel and soil at the toe of slope in order to increase the stability of the downstream slope and at the caved-in spot. The number of siphons was increased to 51 sets.

On the October 31, the reservoir level was at 219.98 m. and the amount of leakage was reduced to about 0.5 m^3/sec.

Eventually, after dumping a lot of material both downstream and upstream and lowering of reservoir level, the leakage water became clear on November 15, with the water at elevation 217.53 m. By November 30, the reservoir was drawn down to elevation 215 m. with the

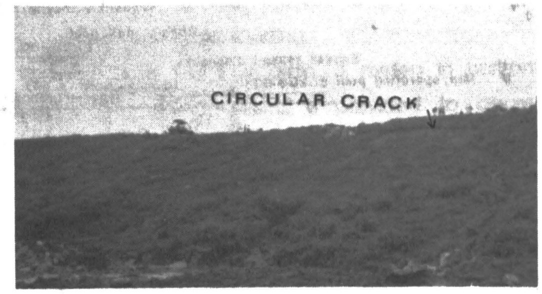

Fig. 5 Circular crack at leakage STA. 0+856

leakage reduced to less than 0.01 m^3/sec.

The dam was finally breached with a cut completely through the dam in the general area of the river channel.

5. POST FAILURE INVESTIGATION

Folllowing the failure, two main hypothesis regarding the causes of leakage were developed.

1) Leakage was initiated through dam embankment.

2) Leakage was initiated through the foundation or contact between the foundation and embankment.

RID had set up a comprehensive investigation program in order to help determine the characteristics and the actual cause of leakage. Various geotechnical investigation and laboratory testing on material samples from the dam and foundation were conducted. The design and construction procedure of the dam, together with all data pertaining to the design such as geological investigation, material investigation and laboratory testings, had been reviewed.

At leakage area, the embankment were carefully stripped out in a sequence of horizontal layers. Inspection and test pits were made to locate any possibility that leakage could have been started from a concentrated leak through defect in the embankment and to collect samples for determination of the embankment material properties.

6. EVALUATION OF PIEZOMETRIC DATA

A number of open-ended standpipe piezometers had been installed in the embankment and foundation at various location and depth to measure pore pressure (Fig.6) . Reading had been made daily since April 1989.

In accordance with the timelines of the responses of piezometers located in the embankment at STA 0+225 and STA 0+700, Elv + 210m indicated no variations in pressures since initial impoundment. While the latter

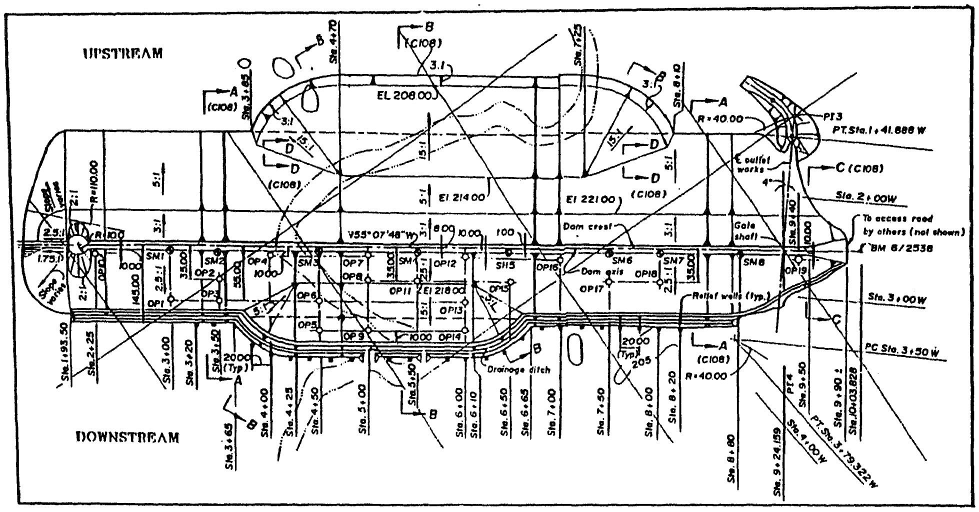

Fig. 6 Plan of Mun Bon Dam

indicated an almost constant response value of about 210.6 m, implying that it was dry, the former indicated an almost constant response value of about 219 m. It might be inferred that seepage through the dam embankment was absent.

Timelines of response of piezometers located in the horizontal fine filter at Elv + 206m, STA 0+500, STA 0+600 and STA 0+800 indicated no variation in seepage pressure since initial impoundment.

Timelines of piezometers located in the foundation at Elv + 190m, Elv + 196m, Elv + 198m, Elv + 200m, and Elv + 201 m, of various chainage appeared normal during reservoir impoundment in 1989 and 1990. In October 1990, during rapid filling of the reservoir, no distress of piezometer response values that may indicate piping, was observed.

Time lag or response lag depended on the soil permeability. A lag in pressure increase was observed in all piezometers. All piezometers were indicating increasing responses while the reservoir elevation had ceased to increase. A greater response lag was observed for piezomters at STA 0+600, Elv + 201m, and STA 0+750, Elv +200 m, which implied that at these locations the foundation was less permeable. Almost identical response lags were observed at STA 0+559, Elv + 190, STA 0+600 and STA 0+650, Elv + 196m, and STA 0+400, Elv + 198, implying that at these locations the foundation had more or less an indentical permeability. A lower response lag was observed for piezometer at STA 0+5000, Elv + 200m indicating that the foundation at this location was more permeable.

7. EVALUATION OF RELIEF WELL DATA

Relief wells were provided along the toe of the dam, at intervals of 20 m. to various depth, from chainage STA 0+320 to STA 0+820 (Fig.7), as an under seepage control measure to alleviate the uplift and seepage forces in the foundation.

A monthly assessment of the performance of the relief well system (April 1989 to October 1990), achieved by plotting representative relief well data across a longitudinal section of the dam, indicated that they were functioning. During rapid filling of the reservoir in October 1990, a substantial increase in relief well level was observed at STA 0+480 (beside location of leakage), (see Fig.7), indicating high seepage through the foundation in the vicinity of this location. The aforesaid observation confirmed results of the geological investigation, which indicated the presence of permeable members that were hydraulically connected across the dam foundation at the leakage locations. Due to the absence of relief wells beyond STA 0+856, it was not possible to determine if similar large underseepage existed in the foundation at this location. However, relief wells in the vicinity at STA 0+800 and STA 0+820 also indicated large underseepage.

8. BORE HOLES AND GEOPHYSICAL INVESTIGATIONS

The soil investigation consisted of a number of bore holes located systematically in a section on the crest as well as on the upstream slope. Four bore holes were located in each of the cross sections where leakages had occurred. In addition, a number of bore holes were made in or near the crest in the neighborhood of the two cross sections.

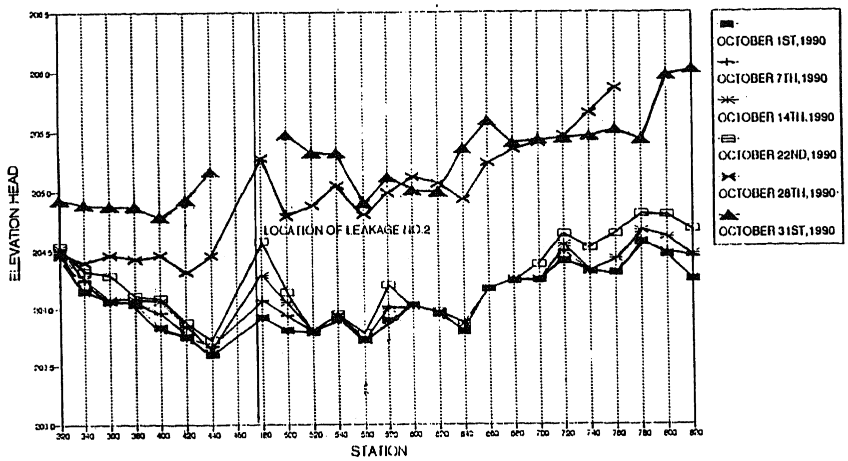

Fig. 7 Relief well record data

The depth of all these bore holes reached at least 2.5 meters into the bedrock material. In the bore holes, Standard Penetration Tests were performed at every meter depth. Further classification tests were carried out on the SPT samples and the permeability of embankment material, alluvial deposits and bedrock were determined in the bore holes.

Based on the data of this soil investigation program in a number of relevant cross sections, geotechnical profiles were drawn up. They confirmed the original description of the foundation materials and did not reveal additional information on the embankment material. However many bore holes near the leakage location, for example at STA 0+848 (see Fig.8), indicated that the embankment was raised on thick permeable sand deposits with thin impervious clay of 1 to 2 meter in between. This thin clay layer might prevent the underseepage water to enter the horizontal blanket drain thus produced excessive uplift pressure against the downstream base of the dam.

In January and March 1991, geo-electrical surveys were carried out along the dam crest and in a few sections longitudinal to both sections of the leakage incidents.

The interpretation of the results of this geophysical survey led to an indication of the presence of holes in an area between about STA 0+785 and STA 0+830 and between about STA 0+855 and STA 0+880. The position of potential holes or cavities were suspected to be below Elv +208 m.

Also, the results indicated anomalies in the foundation material, around STA 0+830 below ELv+198 m. Further anomalies have been detected between STA 0+483 and STA 0+500 at the foundation level.

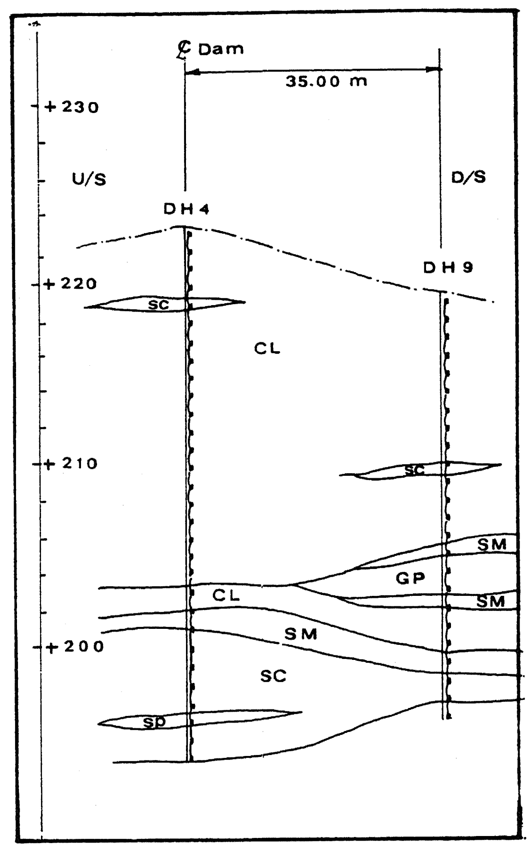

Fig. 8 Geologic Section at STA. 0+848

9. EXCAVATION OF EMBANKMENT AT LEAKAGE LOCATION

Just after the declaration of safe condition of the dam on November 30, 1990. The reservoir was at Elv + 215 m. The dam was then breached to release the remaining impounded water. The cut was made at STA 0+450 near the existing river channel down to the original ground level at Elv +205m. During the excavation only

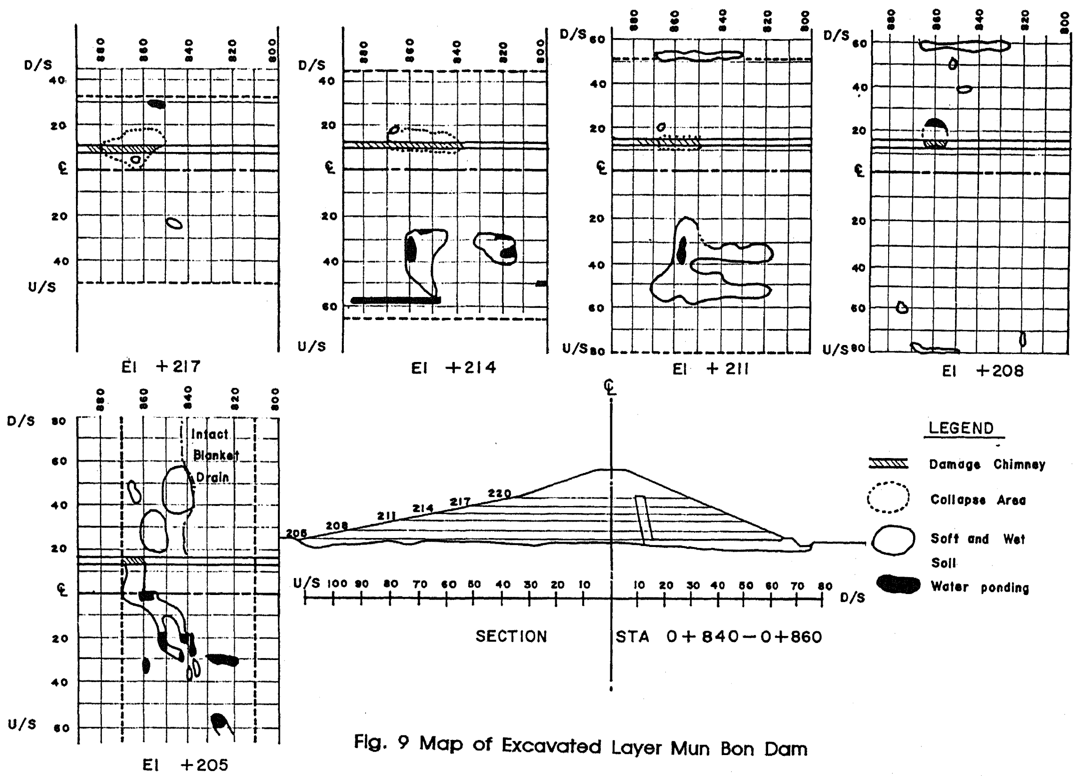

Fig. 9 Map of Excavated Layer Mun Bon Dam

some investigation into the condition of the embankment was performed, since the cut must be completed in time for the coming 1991 rainy season. Therefore no significant evidents related to the leakage was found.

In dry season of 1992, after the completion of all boreholes investigation, a comprehensive investigation of the dam at STA 0+856 was conducted in order to search for any remaining causes of the leakage. The embankment was carefully excavated in a sequence of horizontal layers of 3 meter thick, starting from the remaining surface at approximate Elv +220 m. down to the foundation at Elv +205 m. or lower if possible.

Inspection trenches and test pits were made to determine material properties and locate any leakage features and their relation. For each of the excavation surface the following items were performed, i.e.

- observe and mapping of any unusual or sign of leakage,
- pitting for field density and field permeability tests,
- collect soil sample for various testing.

The result of this excavation was presented in maps as shown in Fig.9. In the figure, maps for each of exposed surfaces of ELV +217, +214, +211, +208 and +205 m. were shown. They indicated the area where wet and soft soil layer, water ponding, and collapse of the embankment were noticed.

From these maps, even though the wet layer and water ponding were found at almost every depth, they were concentrated in the upstream and downstream part of the dam. Care had been made to find the connection of these wet layers from upstream to downstream. Trenches were always excavated just downstream of any wet layers. However, these wet layers would somehow, terminate within a short distance downstream. The embankment downstream of the wet layers gradually changed into intact materials as the excavation was progressed laterally downstream. Overlay map of each elevation, the wet layers and water ponding zones seem to be connected in vertical direction. The only connection from upstream to downstream was at elevation near the foundation, i.e. closed to Elv +205 m.

10. FIELD AND LABORATORY TESTING RESULTS

Permeability of the dam embankment, alluvial foundation and the underlying bedrock, were tested at various boring across the dam.

Field permeability tests of the dam embankment material and the coefficients of per meability from falling-head permeability tests performed on Zone 1 and Zone 7 specimens, extracted from undisturbed block samples, were in the range of 1 x 10^{-7} cm/sec to 1 x 10^{-5} cm/sec, thus suggesting that the embankment was virtually impervious.

Permeability values of the alluvial deposits confirmed the range of values as utilized in the design memoranda, that is 1×10^{-4} cm/sec to 1×10^{-3} cm/sec for sand of channel deposits, and 1×10^{-7} cm/sec to 1×10^{-4} cm/sec for clays of flood basin deposits.

The permeability of the horizontal blanket drain, determined from bore hole at STA 0+838, provided a value of 2.6×10^{-2} cm/sec.

Permeability tests from a number of borings were inferred that bedrock at the dam site beneath the alluvial deposits was semi-impervious, with permeability ranging from 1×10^{-5} cm/sec to 1×10^{-4} cm/sec, however, notwithstanding intense local jointing.

The Zone 1 material, constituting the impervious fill comprised more than half of the volume of the dam embankment. The majority of soil for Zone 1 was classified as CL (Unified Soil Classification System) or inorganic silty-clay of low to medium plasticity, as indicated by the cluster of points above the Casagrande A-line in the plot of liquid limit against plasticity index (See Fig.10). The results of grain size distribution analyses informed that the mean median grain size (D_{50}) was 0.01 mm. and the mean percent clay size content (less than 0.005 mm.) was 44.3%. Fig.11 shows the grain size

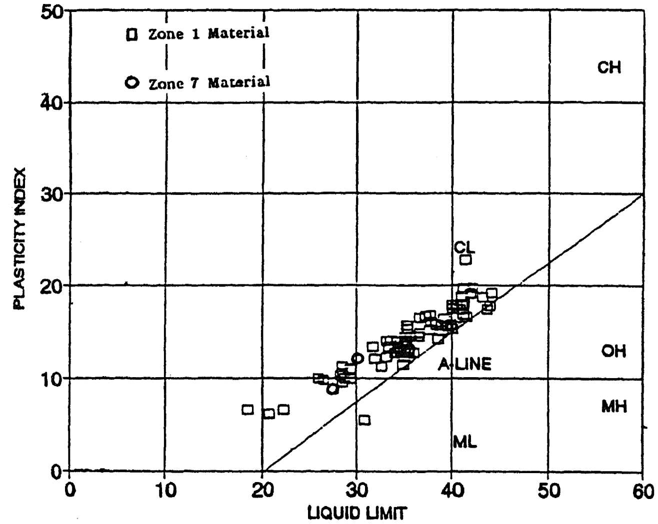

Fig. 10 Plasticity chart of Embankment soil

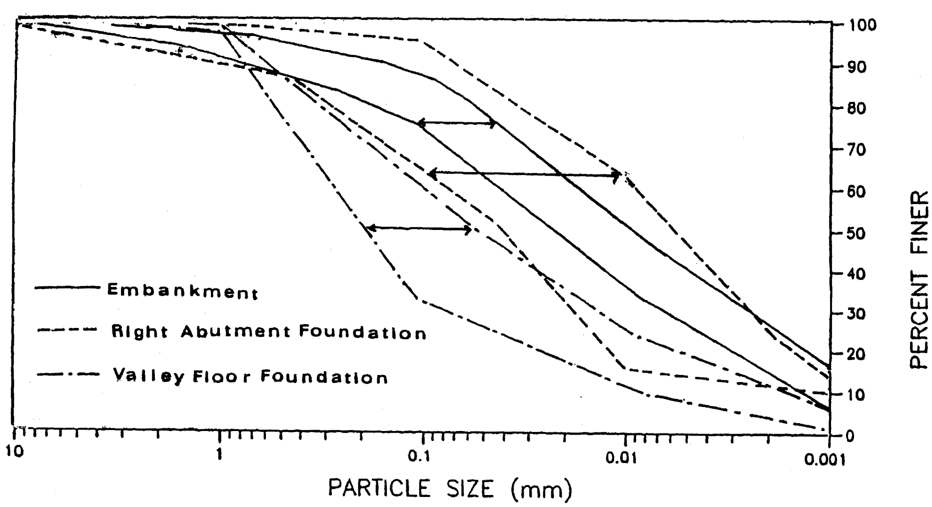

Fig. 11 Grain Size Distribution Range of Embankment and Foundation Materials

distribution range of as-placed Zone 1 material. The material possessed intermediate resistance to piping and high suspectibility to cracking.

Fig.12, comparison of laboratory Proctor density and optimum moisture determination with field density and moisture determination, shows that most materials are compacted on the wet side of the optimum moisture content, nevertheless, within specification limits.

The results of pinhole tests classified Zone 1 material as D1. Values of percent dispersion, obtained from the SCS laboratory dispersion tests were 93% to 64%. Values of turbidity ratio as obtained from the turbidity ratio test were 2 and 1. The Emerson crumb tests classified the material as Grade 4. The results of chemical analyses on the saturation extract as a plot of percent sodium in saturation extract against total dissolved salts in milli-equivalents per litre (meq/1) indicated that the material belonged to Zone 1. All such tests indicated that material belongs to dispersive soil highly susceptible to breaching and tunnel erosion.

The Zone 7 (random fill) material placed along the downstream slope of the dam embankment, were classified as silty-clay of low to medium plasticity (CL). Dispersion results with field observations indicated that the material was also dispersive and susceptible to erosion. The shrinkage cracking and subsequent erosion by rainfall had led to a network of holes and crevices on the entire downstream slope. However, these were confined to the upper 2-3 meter zone.

The Zone 1 and Zone 7 materials were identical in their dispersive behaviour.

Zone 2 was intended to form a chimney drain to safeguard against piping and obliterate seepage through the dam embankment. The influence of the graded horizontal blanket, comprising Zone 2 and Zone 3, was beside ensuring a positive control of the phreatic surface in the dam embankment in the event of high reservoir elevation, to prevent piping and excessive uplift pressure in the foundation.

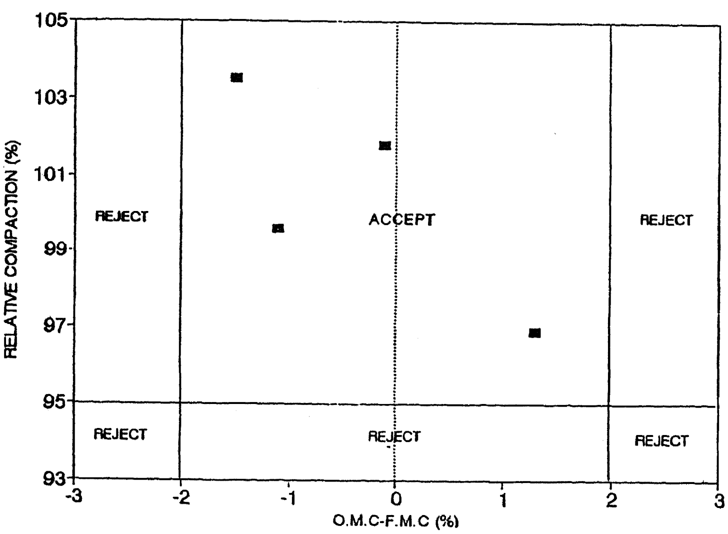

Fig. 12 Plot of Relative Compaction of Zone 1 Material

The results of gradation test on samples of the as-placed sand filter(Zone 2), obtained at STA 0+480, are indicated in Fig.13.

It is evident that wherein grain size distribution of the as-placed sand filter is a uniform fine sand, that gradation is slightly off the specification limits.

The design intension was to use the Zone 2 material to provide head relief and protection against erosion to the foundation materials. However, compatibility of the Zone 2 material and foundation material with respect to filter criteria was not evaluated during design, as was evident from the design memoranda . The design memoranda mentioned that, while the D_{15} and D_{85} of the foundation material beneath the right abutment ranged from 0.0012 mm to 0.0018 mm, and 0.05 mm to 0.10 mm. respectively, that of the foundation material beneath the center and left abutment , ranged from 0.70 mm to 0.15 mm, and 0.28 mm to 0.55 mm respectively. Applying the criteria used in the design of filter and the specified range of sizes for Zone 2 material, it was determined that while the piping criterion (D_{15F} /D_{85B} =5.6) remained unsatisfied for the foundation material beneath the right abutment, the permeability criterion (D_{15F} /D_{15B} =0.67) remained unsatisfied for foundation material beneath the center and left abutment.

Results of post-failure gradation tests on foundation materials beneath the right abutment (See Fig.11) were compared with the gradation of as-placed material. The piping and permeability criteria for filter design were satisfied. Upon performing similar comparisons for left abutment, it was discovered that the filter criteria were satisfied for both foundation material.

11. FEM ANALYSES

A Finite Element Method seepage analysis[3] was independently conducted to simulate seepage condition through the foundation for leakage at STA 0+485 . The analyses assumed that the embankment and bedrock were impervious as compared to the foundation. The objective of the study was to assess the hydraulic interaction between foundation and horizontal drainage blanket at the time of leakage. The coefficient of horizontal permeability for the foundation was assumed to be $1x10^{-3}$ cm/sec based on the results from post-failure tests , and one-sixth of that was used for vertical permeability.

The permeability of the horizontal blanket drain was established iteratively by matching the calculated head with the observed head during the days of the incident. With these assumptions, the analyses showed good agreement within 1 meter difference between calculated and observed heads for most of the piezometer, when the coefficient of permeability of the horizontal blanket drain was 30 times more permeable than that of foundation soil. In other analysis, using the design value for coefficient of permeability for the horizontal blanket drain of 2 cm /sec. which was 2,000 times more permeable than the foundation, yielded only slightly lower head. While in the analysis where the blanket drain was assumed permeability value of 10 times less than the foundation, simulated the clogging of drain, showed a much larger prediction of head in the foundation (See Fig.14). This analysis indicated that no clogging of blanket drain had occurred.

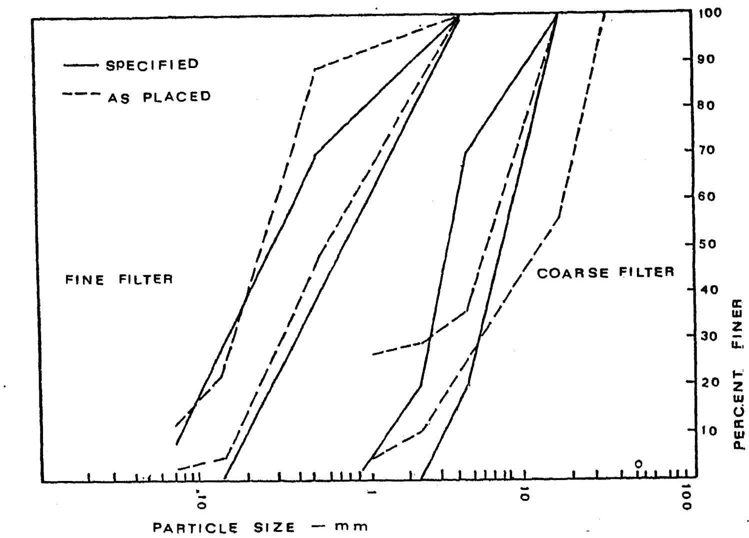

Fig. 13 Gradation Curve for Filter Material

12. POTENTIAL FOR ABANDONED RIVER CHANNEL AT STA 0+856

The nature of deposits and sedimentary structure found in the foundation at Mun Bon Dam is truly representative of 'meandering river' sedimentary environment. The deposits found in such an environment are traditionally divided into two broad categories, namely channel deposits and alluvial deposits. While channel deposits display an upward fining sequence, with sand deposits. The alluvial deposits, comprising flood basin and back swamp deposits, are fine-grained sands, silts,and clays. Often, switching or avulsion of the river channel results in the formation of an abandoned river channel, where the deposits are similar to flood basin deposits. However, abandoned channel deposits are distinguished from flood basin deposits by their channel shaped geometry and by the fact that abruptly pass down into channel floor lag coarse materials. The aforementioned features are concurrent with those observed at Mun Bon Dam, as indicated by the result of post-failure geophysical survey.

The results of the geo-electric investigation are further corroborated by results of the standard penetration tests. Profiles of standard penetration number at boring near the 'meandering river' channel, indicate the presence of loose sediments (N less than 10) to a depth between 5m and 10m. While the presence of loose sediments to a depth of 5m is indicated in the boring near the abandoned river channel around STA 0+856, other boring in the vicinity indicate the presence of medium dense sediments (N greater than 20).

13. MOST PROBABLE MECHANISM IN THE DEVELOPMENT OF LEAKAGE

Based upon details available, the following stages in the development of leakage are suggested.

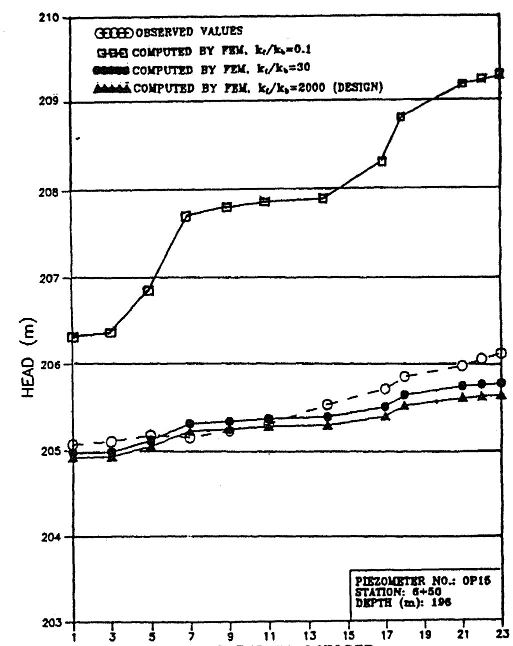

Fig. 14 Comparison between Observed and FEM Pressure Head in Foundation

As the reservoir level rose, between October 4, 1990 and October 22, 1990, a large quantity of water would have gained access to loose and permeable channel deposits, left unexcavated, along the 'meandering' and 'abandoned' river channels. This would created high pressure against the down stream toe of the dam, which would have led to the appearance of leakages at STA 0+856 and STA 0+475.

Loose foundation and embankment material were subsequently removed by the flow, which resulted in the formation of the cavity beneath the embankment. It would be reasonable to assume that first the inclined chimney drain would have collapsed along the wall of the impermeable zone. Since the material of the horizontal drain was already washed away leaving material in the upper part unsupported. At STA 0+475 the caved-in of part of the downstream slope was somehow instrumental in the plugging of the existing leakage hole. Meanwhile, the erosion of foundation and embankment materials enlarged the cavity, causing a further collapse of the leakage cavity wall. To a certain extent, the undermined upstream embankment collapsed such that cracks were developed and provided shorter seepage paths. Enhanced by the dispersive nature of the embankment material, the cracks were quickly developed to sinkholes and resulted in a vortex flow in the vicinity of STA 0+856, observed on October 25, 1990, three days after the first observation of leakage.

Continuous dumping of materials into the holes and the collapse, and the formation of erosion tunnels might have contributed to the fluctuation of leakage quantity at STA 0+856. The continuous leakage led to the erosion of the downstream filter zone, thus generating a slope failure.

14. CONCLUSION

Evidences from post-failure detailed investigation affirmed that the leakage could not have been caused by water seeping into the embankment body from the reservoir through the voids or cracks in the embankment. Thus it should be generally agree that the leakage must initiate from the high permeable foundation.

From the standpoint of fair conclusion, it is necessary also to examine the question: "If the horizontal drainage blanket materials met the design criteria or specification, would leakage still occur ?". It is impossible to form a unique opinion on this question. If the horizontal drainage blanket materials were capable of preventing the erosion of the foundation or embankment as it was intended, it is still not easy to conclude that the dam would never have failed. The materials for horizontal drainage blanket were constructed of a rather uniformed materials throughout the length of the dam. The reservoir pressure subjected to every section of the horizontal blanket drain was almost the same Hence, the possibility for leakage to taking place should be randomly the same everywhere. However, leakage only occurred at two specific locations, i.e. where the foundation composed of river channel deposit.

A more fortunate aspect of this incident was that the leakage started at the time when construction crew and equipments were still station at a dam not very far away. Mobilization of man power and machinery to fight against the failure of the dam was possible then. If the dam had leaked without the aforementioned assistant, the consequences would have been very much more serious.

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